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SOIL INVESTIGATION
RODEO CREEK CONFLUENCE WITH CALABAZAS CREEK

San Jose, California

SEPTEMBER 1974

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INCORPORATED
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File No. A4-0544-J1
September 27, 1974

Mr. Dexter D. Ahlgren
P.O. Box 2293
Sunnyvale, California 94087

Subject: Rodeo Creek Confluence with Calabazas Creek
South of Danridge Drive and East of Blaney Road
San Jose, California
SOIL INVESTIGATION

Dear Mr. Ahlgren:

As you requested, we are pleased to submit herewith the results of our soil investigation performed on Rodeo Creek Confluence with Calabazas Creek on property located north of Danridge Drive and east of Blaney Road in San Jose, California.

The object of our investigation was to determine the soil conditions along the banks of Rodeo Creek where it joins the Calabazas Creek channel and to establish soil criteria for the design of slope retaining structures and erosion control measures for the Rodeo Creek channel. The scope of our investigation was directed towards developing specific recommendations for the stabilizing of the creek banks including erosion control features for the sides and bottom of the creek channel.

A representative from our office visually reviewed the subject property shown in Figure 1, "Site Plan and Location of Test Borings" prior to the test drilling program. With reference to Figure 1, it was found that the bank on the east side of Rodeo Creek, north of the drop structure, had eroded to such a degree that it was beginning to undermine the adjacent private property. The upper bank on the west side was observed to be leveled off and surface erosion had not quite reached the property line of the new residence recently constructed. A deep depression was noted in the unlined Rodeo Creek flow line at the end of the drop structure obviously caused by high velocity flows in the creek channel during the winter time.

Field Investigation

Subsequent to our initial field inspection, seven test borings were drilled on the site on 30 August 1974. The locations of these test borings are shown in Figure 1. The test borings were drilled with 6-inch diameter continuous flight augers and range in depth from 9 to 25 feet below the existing ground surface. The soil profile of all borings is shown in Figures 2 through 8 of this report.

Laboratory Testing

The laboratory testing program was directed towards a determination of the physical and engineering properties of the soils comprising the east and west banks of Rodeo Creek. Tests were performed on soil samples taken from test borings and from the near-surface soils. The shear strength perameters were determined by means of the direct

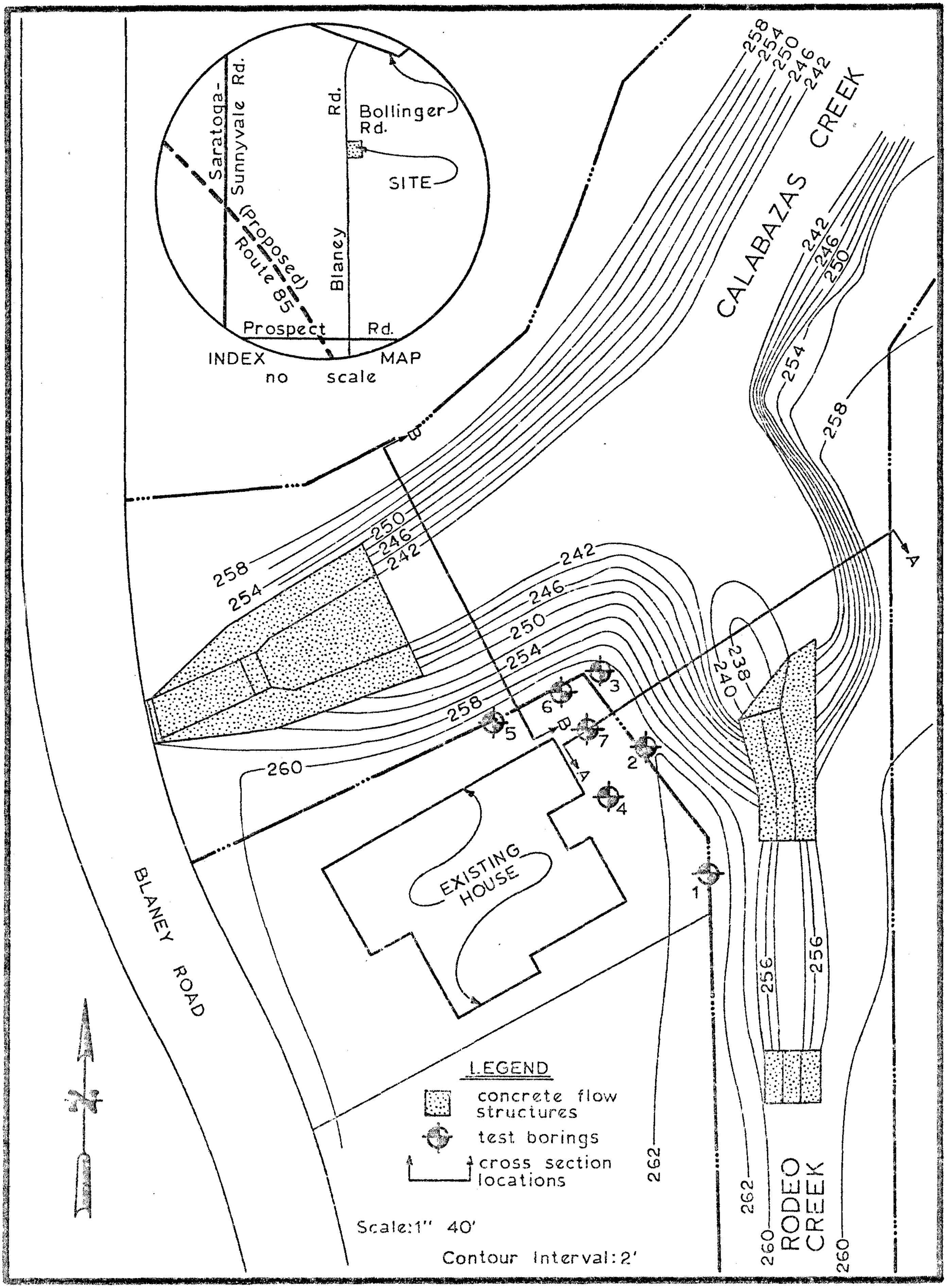


Figure 1 - Site Plan, San Jose, CA

shear tests. All samples to be sheared were then placed in contact with water for a minimum period of 24 hours then allowed to drain briefly before and during the actual shear tests. Specimens were sheared under normal loads ranging from 1000 to 4000 psf. Undisturbed samples were tested for their in-place moisture content and dry unit weight by weighing a known volume of soil before and after drying in the laboratory.

Moisture content and dry density tests were performed on representative undisturbed soil samples in order to determine the consistency of the soil and moisture variations throughout the explored soil profile.

A compaction test was performed in accordance with ASTM Test Designation D1557-70 to determine the moisture-density relationship of the near-surface soils and to obtain the maximum relative compaction and optimum moisture content values of these near-surface soils. The results of all laboratory tests performed are summarized in TABLE I and Figures 2 through 9 of Appendix A.

Soil Conditions

The test borings determined that the west bank of Rodeo Creek is comprised of uncontrolled debris contaminated soil fill over natural ground. The east bank of the creek is made up of natural ground only, with the eroded bank near vertical at the present time. The existing fill material on the west bank varied in thickness between 4 feet and 14-1/2 feet. The natural ground encountered in test borings

1 and 4, below the fill material encountered a stiff brown silty clay layer to a depth of 14 feet. Below this depth in all borings a brown silty sand and coarse gravel was encountered to the bottom of the test borings. Borings 2, 3, 6 and 7 each encountered drilling refusal due to what appeared to be heavy concrete debris in the uncontrolled fill situated over natural ground. The uncontrolled fill is comprised of trash, mixed import soil types, large pieces of concrete and is essentially loose and potentially unstable from a slope stability standpoint.

CONCLUSIONS AND RECOMMENDATIONS

1. The east bank of Rodeo Creek is nearly vertical in configuration due to stream erosion and is comprised of dense undisturbed native soil. No fill exists on this east bank. Any proposed filling done in this side of the channel to reconstruct the creek should be projected at a 2:1 maximum fill slope. Such fills would have to be keyed into the existing creek bottom and into the near vertical natural embankment. Such keying may be difficult due to the vertical condition of the slope, the nearness of erosion to adjacent property lines and the water table below the creek channel. There is a problem that may arise in this possible solution in the fact that any fill material may greatly reduce the channel bottom width of Rodeo Creek on the eastern side of the bank. All such fill should be placed under the supervision of a Soil Engineer and be properly compacted to not less than 90% relative compaction based on the ASTM Laboratory Compaction Test Procedure D1557-70. Fill material should be approved by the Soil Engineer prior to its being transported to the site.

2. The placement of a new structural fill on the west bank of Rodeo Creek, becomes much more difficult. As previously described the west bank is made up of a heterogeneous uncontrolled fill material containing large concrete pieces and debris. The soil portion of the fill is a brown silty sand with gravel and is in a loose to medium dense condition. It is impossible to determine the exact

stability of such debris fill material. Therefore, it is possible that all or a portion of the existing fill is in danger of sliding into the creek channel in the future. Since it appears that the newly constructed fill extends under the existing new house located some 15 to 25 feet back from the top of the fill slope edge the stability of the house foundation is also in question. Its original placement appears to be uncontrolled and not well compacted. If all or a portion of the existing fill is removed and then reconstructed, it might well endanger the existing house. Therefore the reconstruction of this west bank of Rodeo Creek and south bank of Calabazas Creek by means of mass regrading does not appear to be feasible without endangering the new house foundation.

Retaining walls could be constructed at the toe of both the east and west banks. They would have to be designed in such a way as to withstand the lateral forces created by the existing fill and the undisturbed native creek embankments. Such retaining walls could be constructed independent of one another or be tied together at their bases by a concrete slab extending across the bottom of the Rodeo Creek. In the case where they are tied by the channel bottom slab, the slab would create a fully lined "U" shaped concrete channel. The retaining walls would be backfilled to create a level maintenance road in the east bank and a stabile backyard for the existing house on the west side.

4. Such retaining walls should be founded a minimum of 4 feet deep below the lowest adjacent creek bottom and be designed to retain

an equibalent fluid weighing 40 pcf for an "active" earth pressure condition, with horizontal backfill behind the wall. When the backfill behind the wall has a 2:1 slope gradient an "active" earth pressure of 75 pcf (equivalent fluid weight) should be used in the design of the wall. These values do not include any possible surcharge loads such as vehicle loads which may be placed adjacent to the retaining wall. For the passive case, use an equivalent fluid weight value of 250 pcf for compacted native soil or selected structural fill at the base of the wall. A friction factor value of 0.3 (N) may be assumed between the bottom of the retaining wall footings and compacted native soil in designing the retaining wall against sliding. Assume a maximum allowable dead plus live load bearing capacity of 2500 psf for designing the retaining wall footings.

5. The west bank of Rodeo Creek which becomes the south bank of Calabazas Creek appears to have the same loose uncontrolled fill characteristics. Our Boring No. 5 encountered fill material to a depth of 14.5 feet which was the same type of material encountered on the previous borings on the west bank of Rodeo Creek. Therefore, any solution employed for the restoration and protection of the west bank of Rodeo Creek should be incorporated into a redesign and protection of the south bank of Calabazas Creek as it ties into the Rodeo Creek west bank.

6. Gravity bin or reinforced concrete retaining walls appear to be the most suited to the site conditions. Gravity bin walls could

be imbedded below the existing creek channel at least 4 feet and be of such size that they would adequately retain the creek banks. The availability and cost of such construction should be discussed with people supplying them to see which would be the best suited and most economical.

7. Drilled caisson retaining walls with horizontal ties were considered in our investigation, but they do not appear to be well suited for the conditions prevailing at this project site. Horizontal tie back anchors for such caisson walls cannot be drilled through the existing loose debris fill on the west bank or extend horizontally into the adjacent private property on the east bank. In addition the caisson footings would have to be drilled deep into the creek bottom where high ground water and caving sand and gravel deposits exist.

8. The cosmetic type of solution to channel restoration and erosion control such as a gabion rock wall or sack rip-rap installation on the existing fill slopes would not provide the necessary structural stability to the creek banks involved. Should the uncontrolled fill making up the west bank of the creek start to slide, the rip-rap would not be sufficiently strong to resist the sliding and the embankment would fail and the rip-rap would be lost.

9. As we discussed with you in our office, it appears feasible to raise the existing Rodeo Creek channel with compacted soil fill to an elevation which will be higher than the Calabazas Creek channel

and reduce the height of proposed retaining walls on the east and west sides of Rodeo Creek. This would also increase the lateral stability of these Rodeo Creek slope embankments and at the same time reduce costs for retaining wall construction because of their lower heights. Such creek channel filling would have to be performed with approved fill and under the supervision of a Soil Engineer. This method of creek filling in conjunction with the use of retaining walls at the toe of the creek embankments would in our opinion be a good solution to the site problems and would make it unnecessary to remove the existing fill on the west bank of Rodeo Creek. However, it would most likely require some type of retaining structure at the toe of the fill where the fill extends into the edge of Calabazas Creek.

10. No attempt has been made to extablish the point of cutting off construction on Rodeo Creek. It is quite possible that whatever solution is utilized may require extensive reworking of the banks involving Calabazas Creek jurisdiction. It is apparent that at least on the west bank of Rodeo Creek that any solution employed should continue around and meet up with the concrete structure that is existing on Calabazas Creek. The eastern bank of Rodeo Creek may require the extension of its solution to include the present tree on the upper bank and beyond its point into Calabazas Creek to eliminate the obvious erosion that has taken place.

If you have any questions regarding our investigation or the contents of this report please give us a call.

Very truly yours,

APPLIED SOIL MECHANICS, INC.

John M. Bianchi, C.E.

Carl W Greenlee

Carl W. Greenlee, C.E.

sb/eh

Copies: 4 to Addressee

APPENDIX A

Summary of Laboratory Test Results

Logs of Test Borings

Laboratory Compaction Test Plot

TABLE I

Summary of Moisture, Density, Swell and Direct Shear Testing

Sample	Depth	In-Place Conditions		Swell Index	Direct Shear Testing	
No.	feet	Moisture Content (% dry wt.)	Dry Density p.c.f.	% Swell ÷ % Moisture Increase with 100 p.s.f. Surcharge	Angle of Internal Friction (degrees)	Unit Cohesion p.s.f.
1-1	2	14.6	Disturbed			
1-2	5	18.7	105.8			
1-3	10	17.3	112.8			
1-4	19	12.0	118.7			
2-1	2	12.1	Disturbed			
3-1	2	12.1	Disturbed			
4-1	10	22.6	103.0	0.1	13°	
4-2	15	11.7	117.9			
4-3	21	13.7	112.8			
5-1	2	11.7	109.1			
5-2	16	8.9	117.0			

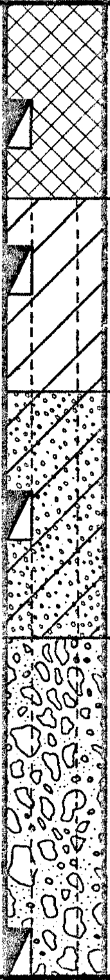
DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	Penetration Resistance Blows/ft	DESCRIPTION	IN-PLACE DRY DENSITY p.c.f.	IN-PLACE MOISTURE CONTENT % dry wt.
				Boring No. 1		
0 2	1-1			Brown Silty SAND (uncontrolled fill) and gravel plus debris, dry, med. dense	Disturbed	14.6
4 6	1-2		30	Dark Brown Silty CLAY (natural ground) moist, very stiff	105.8	18.7
8 10 12	1-3		24	Tan Brown Silty CLAY with fine gravel, moist very stiff	112.8	17.3
14 16 18	1-4		46	Tan Brown Silty SAND and coarse gravel, moist, dense	118.7	12.0
20				Boring Terminated at 20 Feet		

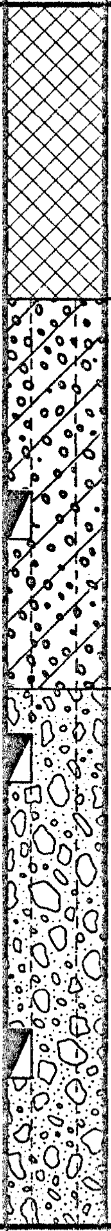
Figure 2 - Log of Test Boring No. 1

DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	Penetration Resistance Blows/ft	DESCRIPTION	IN-PLACE	
					DRY DENSITY p.c.f.	MOISTURE CONTENT % dry wt.
0				Boring No. 2		
2	2-1	▨	6	Brown Silty SAND and gravel (uncontrolled fill) mixed with debris, dry, loose	Disturbed	12.1
4		▨				
6		▨				
8		▨		Refusal to drilling		
10				Boring Terminated at 10 Feet due to large concrete pieces in fill.		

Figure 3 - Log of Test Boring No. 2

DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	Penetration Resistance Blows/ft	DESCRIPTION	IN-PLACE	
					DRY DENSITY p.c.f.	MOISTURE CONTENT % dry wt.
				Boring No. 3		
0 2 4 6 8	3-1	▨	12	Brown Silty SAND and gravel (uncontrolled fill) dry, mixed with debris, loose to moderately dense Drilling Refusal	Disturbed	12.1
9				Boring Terminated at 9 Feet due to large concrete pieces in fill.		

Figure 4 - Log of Test Boring No. 3



DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	Penetration Resistance Blows/ft	DESCRIPTION	IN-PLACE	
					DRY DENSITY p.c.f.	MOISTURE CONTENT % dry wt.
				Boring No. 4		
0				Brown Silty SAND and gravel (uncontrolled fill) mixed with debris, dry, loose to moderately dense		
2						
4						
6				(natural ground)		
8						
10	4-1		18	Dark Brown Silty CLAY with gravel, moist very stiff	103.0	22.6
12						
14	4-2		14		117.9	11.7
16				Brown Silty SAND and gravel, moist, medium dense		
18						
20	4-3		26	Becoming more dense	112.8	13.7
22						
24						
25				Boring Terminated at 25 Feet		

Figure 5 - Log of Test Boring No. 4

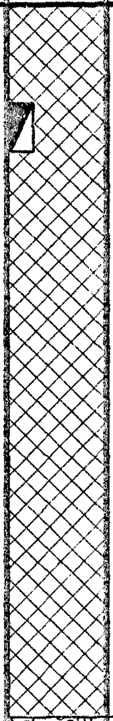
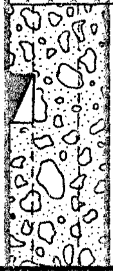
DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	Penetration Resistance Blows/ft	DESCRIPTION	IN-PLACE DRY DENSITY p.c.f.	IN-PLACE MOISTURE CONTENT % dry wt.
				Boring No. 5		
0 2 4 6 8 10 12 14	5-1			Brown Silty SAND (uncontrolled fill) and gravel mixed with debris, dry, loose to medium dense	109.1	11.7
16 18	5-2		29	(natural ground) Brown Silty SAND and gravel moist, dense	117.0	8.9
20				Boring Terminated at 20 Feet		

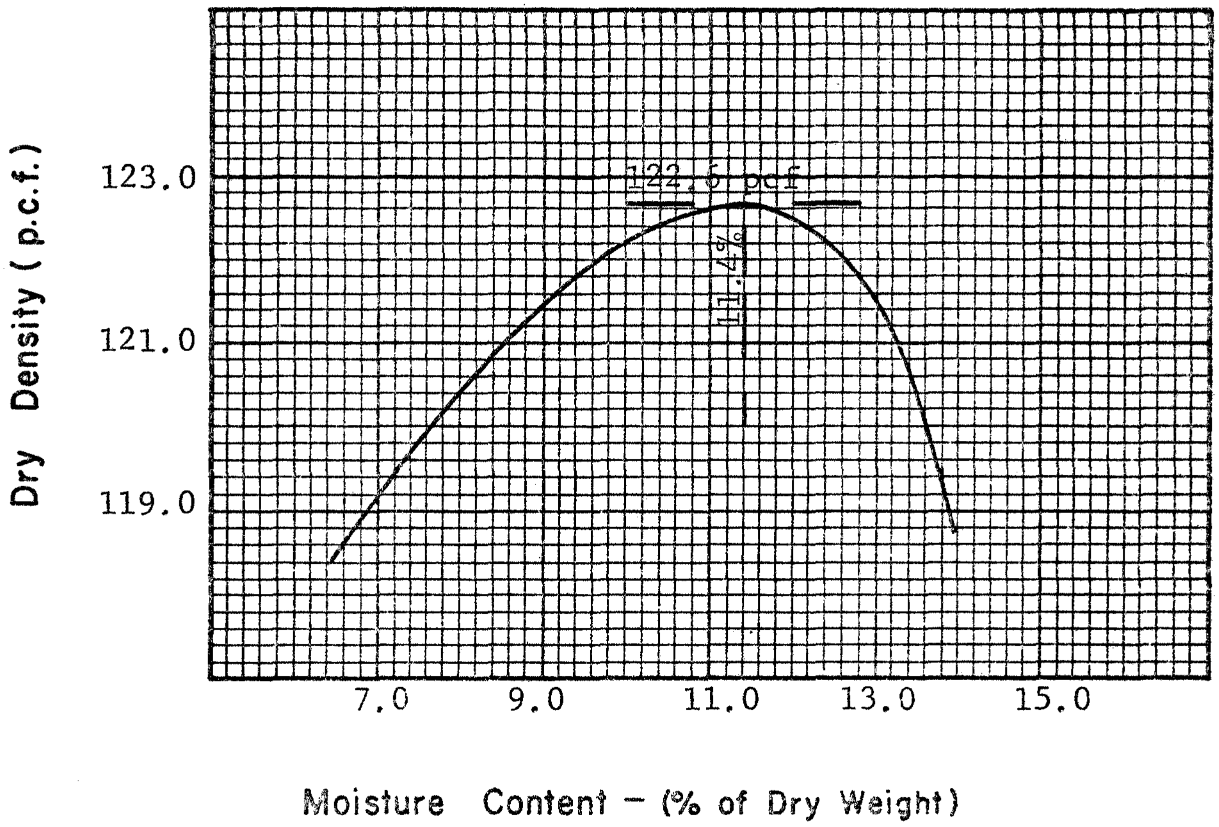
Figure 6 - Log of Test Boring No. 5

DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	Penetration Resistance Blows/ft	DESCRIPTION	IN-PLACE	
					DRY DENSITY p.c.f.	MOISTURE CONTENT % dry wt.
				Boring No. 6		
0 2 4 6 8 10 12 14 16		▨		Brown Silty SAND and gravel (uncontrolled fill) mixed with debris, dry, loose to medium dense		
17				Boring Terminated at 17 Feet due to large concrete pieces in fill.		

Figure 7 - Log of Test Boring No. 6

DEPTH IN FEET	SAMPLE NO.	LOG & LOCATION OF SAMPLE	Penetration Resistance Blows/ft	DESCRIPTION	IN-PLACE DRY DENSITY p.c.f.	IN-PLACE MOISTURE CONTENT % dry wt.
				Boring No. 7		
0 2 4 6 8		▩		Brown Silty SAND and gravel (uncontrolled fill) mixed with debris, dry, loose to medium dense		
10 12 14 16 18		▨		(natural ground) Brown Silty CLAY moist, stiff		
		⁘		Brown Silty SAND and gravel moist, dense		
20				Boring Terminated at 20 Feet		

Figure 8 - Log of Test Boring No. 7



Sample : From Test Boring No. 1 at 2 Feet Below Ground Surface

Description : Uncontrolled Fill Material-Brown Silty SAND with Gravel

Laboratory Test Procedure : ASTM 1557-70

Maximum Dry Density : 122.6 pcf

Optimum Moisture Content : 11.4%

Figure 9 - Laboratory Compaction Curve